CHAPTER 8

LINTELS

8-1. Introduction. A lintel is a horizontal beam supporting loads over an opening. This chapter covers the design of reinforced masonry lintels. Reinforced masonry lintels must have all cores and other voids solidly grouted. Precast reinforced concrete or structural steel lintels will be designed in accordance with ACI 318 and the AISC Steel Construction Manual, respectively, except the deflection limits contained in this chapter will be followed. Torsion is not covered in this chapter. Where torsion is a major consideration, the designer should consider precast reinforced concrete lintels with closed loop stirrups. The principles of this chapter may be used for designing beams of reinforced masonry that are not lintels. Except as contained herein, design criteria, section properties, material properties, design equations, and allowable stresses are contained in chapter 5.

8-2. Loading. In addition to its own weight, a lintel may carry distributed loads from above, both from the wall weight and from floor or roof framing. The lintel may also carry concentrated loads from the framing members above.

a. Distributed loads. The shape of the loading diagram for the distributed loads to the lintel depends upon whether arching action of the masonry above the opening can be assumed. When arching action occurs, the lintel supports only the masonry that is contained within a triangle having sides which begin at the ends of the lintel and slope upward and inward 45 degrees from the horizontal to converge at an apex above the center of the lintel. See figure 8-1 for an illustration of this triangular lintel loading distribution. When the lintel deflects into the opening over which it spans, the masonry above the triangle will arch over the lintel and be supported by the more rigid walls on either side of the opening. For the arch to be stable, both ends of the opening must have sufficient horizontal restraint to provide the confining thrust necessary to support it laterally. Therefore, arching action should not be considered where the end of the arch and the lintel are near a wall corner, near a control or building expansion joint, or in stacked bond walls. When arching action can be assumed, the lintel will be designed to carry its own weight plus the weight of masonry within the triangle above. Where uniform floor or roof loads are applied to the wall above the apex of the triangle, it will be assumed that arching action will carry these loads around the opening and not load the lintel. When uniform floor or roof loads are applied below the apex of the triangle, arching action cannot take place and these loads will be carried downward and applied uniformly on the lintel. Also, when a uniform floor or roof load is applied below the apex of the triangle, it will be assumed that all of the weight of the masonry above the lintel is uniformly supported by the lintel.

b. Concentrated loads. Concentrated loads from beams, girders, or trusses framing into the masonry wall above an opening will be distributed downward from the apex of a triangle which is located at the point of load application. The sides of this triangle make an angle of 60 degrees with the horizontal. The load is transferred as a uniform load over the base of the triangle. This uniform load may extend over only a portion of the lintel. See figure 8-1 for an illustration of the distribution of concentrated loads on lintels.

8-3. Allowable deflection. For all lintels, the total deflection will be limited to L/600, not to exceed 0.3 inches.

8-4. Masonry lintel deflections.

a. Deflection parameters. When calculating lintel deflections the following parameters will be used.

(1) Span length. The assumed span for deflection calculations will be the distance between the centers of supports, illustrated as dimension “L” on figure 8-1.

(2) Moment of Inertia. The moment of inertia for deflections will be the effective moment of inertia, I_e, which will be determined as follows:

$$I_e = (M_{cr}/M_{max})^3 I_g + [1 - (M_{cr}/M_{max})^3] I_{cr} \text{ (in}^4\text{)} \qquad \text{(eq 8-1)}$$

Where:

M_{max} = The maximum moment in the member at the design load level, inch-pounds.

M_{cr} = The moment that causes flexural cracking of the lintel section, given in equation 8-2, inch-pounds.

$$M_{cr} = \frac{f_r I_g}{Y_t} \qquad \text{(eq 8-2)}$$

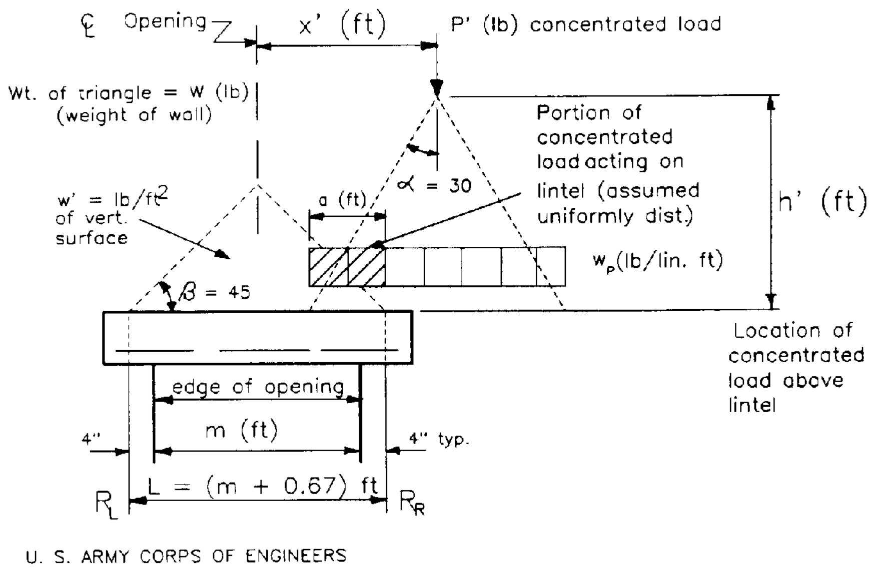

Figure 8–1. Triangular and Concentrated loadings on lintels.

Where:

f_r = The modulus of rupture, which is provided in chapter 5.
Y_t = The distance from the compression face to the neutral axis of the lintel, inches.
I_g = The moment of inertia of the uncracked lintel cross section about the centroid, in^4.

$$I_g = \frac{b(h^3)}{12}$$

b = The actual width of the lintel, inches.
h = The total depth of the lintel, in.
I_{cr} = The moment of inertia of the cracked section of the lintel, in^4.

$$I_{cr} = \frac{b(kd)^3}{3} + nA_s(d - kd)^2 \ (in^4) \qquad \text{(eq 8–3)}$$

Values of I_{cr} for most masonry lintels are provided in the tables in appendix B. For continuous members, the effective moment of inertia may be taken as the average of values obtained from equation 8-1 for the critical positive and negative moment sections. Ie will not be assumed greater the I_g.

b. Total deflection (long term). For masonry lintels, the total deflection, Δ_{mt}, will be the deflection due to short term loadings, Δ_{ms}, (such as live loading and transient dead loadings) combined with the deflections due to long term dead loadings, Δ_{ml}, as follows:

$$\Delta_{mt} = \Delta_{ms} + (3)(\Delta_{ml}) \ \text{(inches)} \qquad \text{(eq 8-4)}$$

8-5. Bearings pressure at lintel reaction. The minimum bearing length will be eight inches. The minimum bearing area, A_{brg}, will be:

$$A_{brg} = 8b \ (in^2) \qquad \text{(eq 8-5)}$$

Fully grouted cores are required below the lintel bearing area. It is reasonable to assume a triangular stress distribution when determining the maximum bearing stress, f_{brg}. When this assumption is made, the maximum bearing stress occurs at the face of the support and is determined by the following equation:

$$f_{brg} = \frac{(2)(R_{brg})}{A_{brg}} \ \text{(psi)} \qquad \text{(eq 8–6)}$$

Where:

R_{brg} = The end reaction of the lintel, lbs.

The maximum bearing stress will not exceed the allowable bearing stress, which is given in chapter 5. If the lintel is restrained against rotation at the support, a uniform stress distribution will be assumed.

8-6. Lateral support. When the tops of lintels are at the tops of walls or when the provisions of this chapter are used to design concrete masonry beams other than lintels, the compression face of the lintel or beam must be given lateral support. The clear distance between points of lateral support of the compression face will not exceed 32 times the least width of the compression face.

8-7. Design aids. Appendix C provides design tables to aid the designer in designing masonry lintels.

8-8. Design examples.

a. Design example 1.

(1) *Given.*

(a) 8-inch CMU nonloadbearing wall

(b) Wall height = 12 feet

(c) f'_m = 1350 psi

(d) $F_m = 1/3f'_m$ = 450 psi

(e) E_m - $1000f'_m$ - 1,350,000 psi

(f) Type S mortar

(g) Reinforcement

f_y = 60,000 psi

F_s = 24,000 psi

E_s = 29,000,000 psi

(h) $$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$$

(i) A door opening 3'-4" wide by 7'-4" high is located in the wall as shown in figure 8-2.

(2) *Problem.* Design the lintel over the door.

(3) *Solution.* Due to the location of the lintel within the wall panel, the confining end thrust necessary to provide arching action may be assumed. Therefore, the lintel must support its own weight plus the weight of the triangle of masonry above the door and below the arch. Assume an 8-inch by 8-inch CMU lintel.

Flexural Check

(a) Determine the maximum moment due to the loading, M_{max}, as follows:

$$M_{max} = \frac{wL^2}{8} + \frac{w'L^3}{24}$$

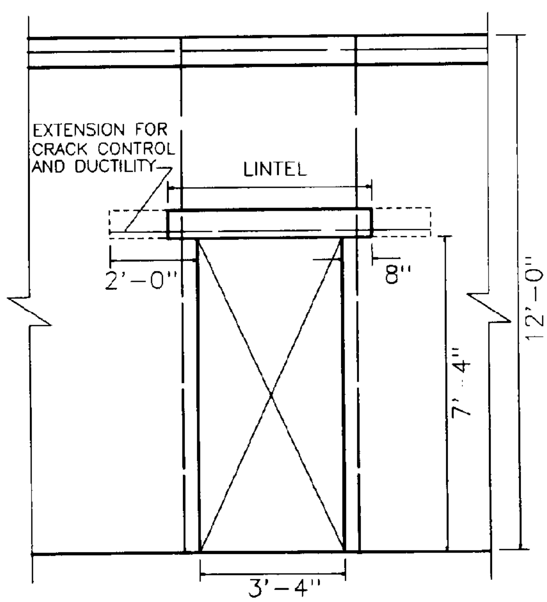

Figure 8–2. Example 1 wall elevation.

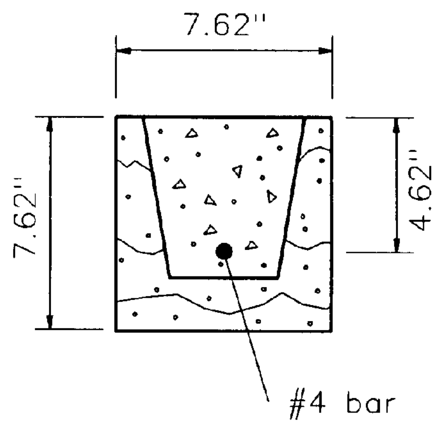

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Figure 8–3. Example 1 lintel cross section.

Where:

w = The lintel weight = 62 lb/ft

w' = The unit weight of the masonry triangle (assume no reinforced filled cells) = 50 psf

$$M_{max} = \frac{(62)(4.0)^2}{8} + \frac{(50)(4.0)^3}{24} = 257 \text{ ft-lb}$$

(b) Determine the area of reinforcement, A_{sb}, required to provide a balanced steel ratio, P_e.

$A_{sb} = (P_e)(b)(d)$

Where:

P_e=0.0027 (See table 5-9)

b = The lintel width = 7.62 in

d = The effective depth of lintel = 4.62 in

$A_{sb} = 0.0027(7.62)(4.62) = 0.095 \text{ in}^2$

(c) The minimum reinforcement required above any wall opening is 1-#4 bar, $A_s = 0.20 \text{ in}^2$. $A_s = 0.20 \text{ in}^2 > A_{sb} = 0.095 \text{ in}^2$ therefore; the design section (shown in figure 8-3) is over-reinforced and the compressive stress in masonry will control over the tensile stress in the reinforcement.

(d) The masonry resisting moment, $M_{rm,}$ is determined as follows:

$$M_{rm} = \frac{F_m kjbd^2}{2(12)} \text{ ft-lb}$$

Where:

$$k = \left[(np)^2 + 2np \right]^{½} - np$$

And;

$$p = \frac{A_s}{bd} = \frac{0.20}{7.62 \times 4.62} = 0.00568$$

So;

$k = [(21.5 \times 0.00568)^2 \pm 2(21.5 \times 0.00568)]^{½}$
$- (21.5 \times 0.00568) = 0.387$

$j = 1 - k/3 = 1 - (0.387/3) = 0.871$

$$M_{rm} = \frac{(450)(0.387)(0.871)(7.62)(4.62)^2}{2(12 \text{ in/ft})} = 1028 \text{ ft-lb}$$

$M_{max} = 257 \text{ ft-lb} < M_{rm} = 1028 \text{ ft-lb}$

...Flexure O.K.

Deflection Check

(a) Determine the moment that causes flexural cracking of the lintel section, M_{cr}, as follows:

$$M_{cr} = \frac{f_r I_g}{Y_t} \text{ ft-lb}$$

Where:

$f_r = 2.5\sqrt{f'_m} = 2.5\sqrt{1350}$ psi = 91.8 psi

$I_g = (7.62)(7.62)^3/12 = 281$ in^4

$Y_t = 7.62/2 = 3.81$ in

$$M_{cr} = \frac{(91.8)(281)}{3.81(12 \text{ in/ft})} = 564 \text{ ft-lb}$$

Since $M_{cr} > M_{max}$;the lintel is not cracked, therefore I_g is used in lieu of I_e.

(b) Since all the loadings are long term (dead load), the total lintel deflection, Δ_{mt}, is determined by modifying equation 8-4 as follows:

$$\Delta_{mt} = (3)(\Delta_{ml})$$

$$= (3)\left[\frac{5wL^4}{384EI} + \frac{WL^3}{60EI}\right]$$

$$\Delta_{mt} = (3)\left[\frac{5(62)(4.0)^4(1728 \text{ in}^3/\text{ft}^3)}{384(1,350,000)(281)} + \frac{(200)(4.0)^3(1728 \text{ in}^3/\text{ft}^3)}{60(1,350,000)(281)}\right] = 0.0057 \text{ in}$$

(c) Determine the allowable lintel deflection, Δ_{allow}, and compare to the maximum lintel deflection, Δ_{max} = 0.30 in, as follows;

$$\Delta_{allow} = \frac{L}{600} = \frac{(4)(12 \text{ in/ft})}{600} = 0.08 \text{ in}$$

$$\Delta_{allow} < \Delta_{max};\ \text{Use } \Delta_{allow} = 0.08 \text{ in}$$

$$\Delta_{allow} = 0.08 \text{ in} > \Delta_{mt} = 0.0057 \text{ in}$$

...Deflection O.K.

Shear Check

(a) Determine the shear loading, V, as follows:

$$V = \frac{wL}{2} + \frac{W}{2}$$

Where:

W = The weight of the triangular shaped wall segment, lbs.
= (50psf)(4.0ft)(2.0ft)/2 = 200 lbs

$$V = \frac{(62)(4.0)}{2} + \frac{200}{2} = 224 \text{ lb}$$

(b) Determine the shear stress in the lintel due to loading, as follows:

$$f_v = \frac{V_{max}}{bd} = \frac{224}{(7.62)(4.62)} = 6.36 \text{ psi}$$

(c) Determine the allowable shear stress, F_v, as follows:

$F_v = F_v = 1.0\sqrt{f'_m} = (1.0)\sqrt{1350}$ psi = 36.7 psi

$F_v = 36.7 > f_v. = 6.36$ psi

...Shear O.K.

Bearing Check

(a) Determine the maximum bearing stress, $f_{brg(max)}$, assuming a triangular stress distribution as follows:

$$f_{brg(max)} = \frac{V_{max}}{A_{brg}}$$

Where:

A_{brg} = The bearing area of the lintel, in^2
= 8 in × 7.62 in = 61 in^2

$$f_{brg(max)} = \frac{2(224)}{61} = 7.3 \text{ psi}$$

(b) Determine the allowable bearing stress, F_{brg}, as follows:

$F_{brg} = 0.25(f'_m)$
= 0.25(1.350) psi = 338 psi

$F_{brg} = 338$ psi $> f_{brgmax} = 7.3$ psi

...Bearing O.K.

(4) Summary: The 8-inch x 8-inch CMU lintel reinforced with 1-#4 bar is sufficient.

b. Design example 2.

(1) *Given.*

(a) 8-inch CMU loadbearing wall
(b) Wall height = 14 ft
(c) Uniform dead load (w_{DL}) = 100 plf
(d) Uniform live load (w_{LL}) = 250 plf
(e) Concentrated live load (P') = 5000 lbs
(f) f'_m = 1350 psi
(g) $F_m = 1/3f'_m$ = 450 psi
(h) $E_m = 1000f'_m$ = 1,350,000 psi
(i) Type S mortar
(j) Reinforcement:
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi

(k) $$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$$

(l) A door 12 ft wide by 10 ft high is located in the wall as shown in figure 8-4.
(m) Assume the wall above the lintel is reinforced vertically at 32 in o.c.
(n) There is a continuous 8 inch bond beam at the top of the wall.
(o) The concentrated live load, P', is located 7 ft to the right of the centerline of door.

(2) *Problem*. Design the lintel to support the given dead and live loadings.

(3) *Solution*. Since the loading is applied below the apex of the triangle (see figures 8-1 and 8-4), arching action cannot be assumed. The lintel must be designed for the full applied dead and live loading above.

Lintel Depth Determination.

(a) The lintel depth will be determined so that shear reinforcement is not required. To establish the dead loading, assume a lintel depth of 24 inches.

(b) The lintel loadings are as follows:

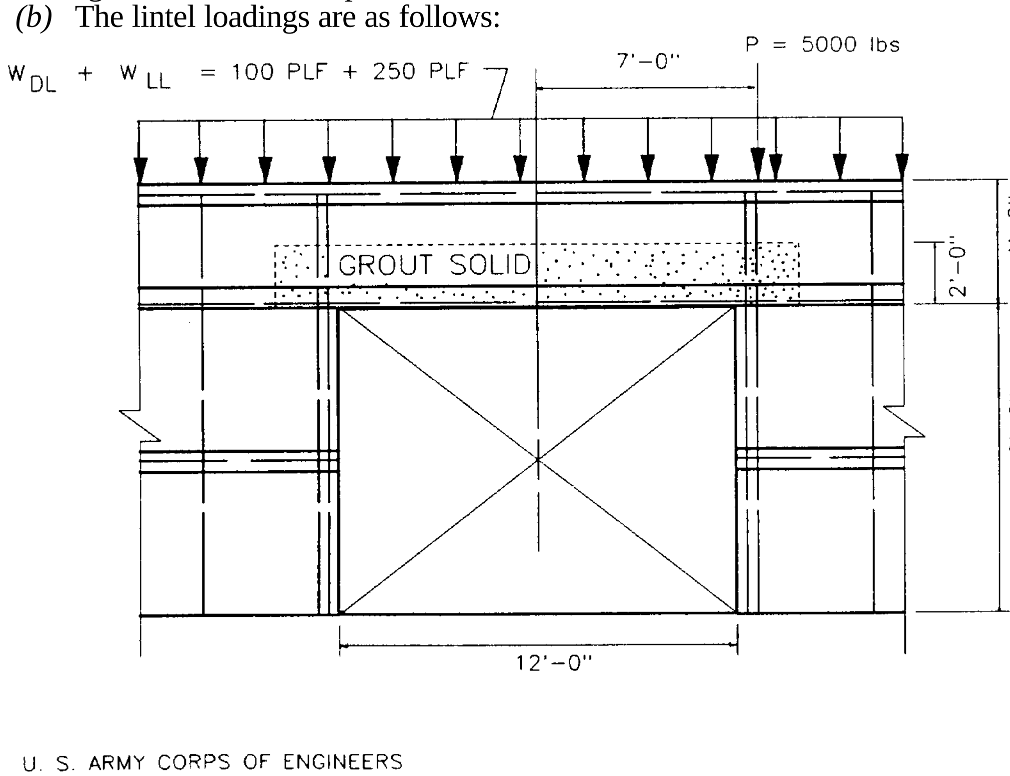

Figure 8–4. Example 2 wall elevation.

Design dead load is "w". Unit weights of 8-inch wall are; solidly grouted, 92 psf; and grouted at 32 inches on center, 65 psf.

$w = w_{DL} + 2_{lin} + w_{wall}$

Where:

w_{lin} = the weight of the lintel, lbs/ft
= (92 psf)(2.00 ft) = 184 plf

w_{wall} = (92 psf)(0.67 ft) + (65 psf)(1.33 ft)
= 62 plf + 86 plf = 148 plf

w = 100 + 184 + 148 = 432 plf

Design uniform live load is w_{LL}.

w_{LL} = 250 plf

Uniform distribution of the concentrated live load on the lintel, w_p, is determined as follows: (See figure 8-1 for an explanation of the terms used in this distribution.)

$$w_p = \frac{P'}{2h'\tan\alpha} = \frac{P'}{1.155h'} = \frac{5000}{1.155(2.0)} = 2165 \text{ plf}$$

And;

a = (h'tan α + 0.5L - 0x')
= (0.577W + 0.5L - x')
= [(0.577 × 2.0) + 0.5(12.67) - 7.0)]
= 0.49 ft

(c) Determine the shear loading, V, as follows:

$$V = \frac{w_{LL}L}{2} + \frac{wL}{2} + \frac{w_p a(2L - a)}{2L}$$

Where:

L = The design span length of the lintel, feet.
= 12.00 ft ± 0.67 ft = 12.67 ft

$$V = \frac{(250)(12.67)}{2} + \frac{(432)(12.67)}{2} + \frac{(2165)(0.49)[(2 \times 12.67) - 0.49)]}{2(12.67)} = 5361 \text{ lbs}$$

(d) Minimum lintel depth without shear reinforcement, d_{reqd}, is determined as follows:

$$d_{reqd} = \frac{5361}{(37)(7.62)} = 19.01 \text{ in}$$

Where:

F_v = The allowable shear stress, psi.
= $1.0\sqrt{f'_m} = \sqrt{1350}$ lb/in^2 = 37 lb/in2

b = the actual lintel width, in.
= 7.62 in

$$d_{reqd} = \frac{5361}{(37)(7.62)} = 19.01 \text{ in}$$

For a 24 inch deep lintel, the actual effective beam depth, d_{act}, is 20.62 inches.

d_{act} = 20.62 in > d_{reqd} = 19.01 in

...24-Inch Lintel Depth O.K.

Flexural Check

(a) Determine the maximum moment due to loading, M_{max}, as follows:

$$M_{max} = \frac{w_{LL}L^2}{8} + \frac{w_{DL}L^2}{8} + \frac{w_p a^2}{4}$$

$$= \frac{(250)(12.67)^2}{8} + \frac{(432)(12.67)^2}{8} + \frac{(2165)(0.49)^2}{4}$$

= 13,814 ft-lb

(b) Determine the area of reinforcement, A_{sb}, required to provide a balanced steel ratio, p_e.

$A_{sb} = (p_e)(b)(d)$

Where:

p_e = 0.0027 (See table 5-9)
b = The lintel width = 7.62 inches
d = The effective depth of lintel = 20.62 inches

$A_{sb} = 0.0027(7.62)(20.62) = 0.424 \text{ in}^2$

(c) The minimum reinforcing steel required above any wall opening is 1-#4 bar, $A_s = 0.20 \text{ in}^2$.

$A_s = 0.20 \text{ in}^2 < A_{sb} = 0.424 \text{ in}^2$

Try 2-#4 bars ($A_s = 0.40 \text{ in}^2$) as shown in figure 8-5.

(d) The masonry resisting moment, M_{rm}, is determined as follows:

$$M_{rm} = \frac{F_m kjbd^2}{2(12)}$$

Where:

$$k = \left[(np)^2 + 2np \right]^{1/2} - np$$

And:

$$p = \frac{A_s}{bd} = \frac{0.40}{(7.62)(20.62)} = 0.0025$$

$$k = \left[(21.5 \times 0.0025)^2 + 2(21.5 \times 0.0025) \right]^{1/2} - (21.5 \times 0.0025)$$
$$= 0.28$$

$$j = 1 - k/3 = 1 - (0.28/3) = 0.906$$

$$M_{rm} = \frac{(450)(0.28)(0.906)(7.62)(20.62)^2}{2(12 \text{ in/ft})}$$

...O.K.

(e) The reinforcing steel resisting moment, M_{rs}, is determined as follows:

$$M_{rs} = \frac{F_s A_s jd}{12}$$
$$= \frac{(24{,}000)(0.4)(0.906)(20.62)}{12 \text{ in/ft}} = 14{,}945 \text{ ft-lb}$$

$M_{rs} = 14{,}945 \text{ ft-lb} > M_{max} = 13{,}814 \text{ ft-lb}$ ...Flexure O.K.

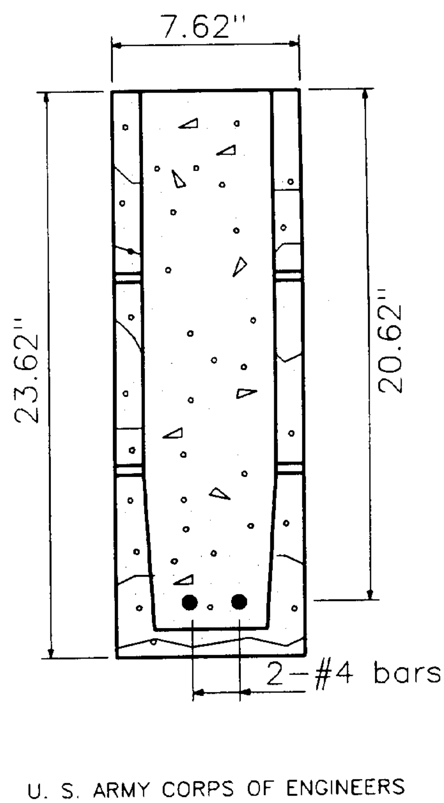

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Figure 8-5. Example 2 lintel cross section.

Note: Steel governed the design. Also, note Mrm and Mrs could have been taken from appendix C, table C-9.

Deflection Check

(a) Determine the moment that causes flexural cracking of the lintel section, M_{cr}, as follows:

$$M_{cr} = \frac{f_r I_g}{Y_t}$$

Where:

f_r = $2.54\sqrt{f'_m} = 2.5\sqrt{1350}$ lb/in^2 = 91.8 lb/in^2

I_g = $(7.62)(23.62)^3/12 = 8368$ in^4

Y_t = $23.62/2 = 11.81$ in

$$M_{cr} = \frac{(91.8)(8368)}{(11.81)(12 \text{ in/ft})} = 5420 \text{ ft-lb}$$

Since $M_{cr} < M_{max}$; the lintel is cracked, therefore the effective moment of inertia, Ie, must be computed as follows:

$$I_e = (M_{cr}/M_{max})^3 I_g + [1 - (M_{cr}/M_{max})^3]\, I_{cr}$$

Where:

I_{cr} = The moment of inertia of the cracked section, in^4

$$I_{cr} = \frac{b(kd)^3}{3} + nA_s(d - kd)^2 \text{ in}^4$$

$$= \frac{7.62(0.28 \times 20.62)^3}{3} + 21.5(0.4)[20.62 - (0.28)(20.62)]^2$$

$$= 2384 \text{ in}^4$$

Note. I_{cr} could be taken from table C-9.

$$I_e = \left[\frac{5420}{13{,}814}\right]^3 (8368) + \left[1 - \left[\frac{5420}{13{,}814}\right]^3\right] (2384) = 2745$$

(b) Determine the lintel deflection, m_t, as follows:

$$\Delta_{mt} = \Delta_{ms} = (3)(\Delta_{ml})$$

$$= \frac{5w_{LL}L^4}{384EI} + (3)\left[\frac{5w_{DL}L^4}{384EI}\right]$$

Since the line of action of the concentrated load is located off the lintel span and the vertical height of the distribution triangle is only 2 feet, the effects of the concentrated load on the centerline deflection is negligible and will be ignored.

$$\Delta_{mt} = \frac{5(250)(12.67)^4(1728 \text{ in}^3/\text{ft}^3)}{384(1{,}350{,}000)(2745)}$$

$$+ (3)\left[\frac{5(432)(12.67)^4(1728) \text{ in}^3/\text{ft}^3)}{384(1{,}350{,}000)(2745)}\right] = 0.242 \text{ in}$$

(c) Determine the allowable lintel deflection, Δ_{allow}, and compare to the maximum lintel deflection, Δ_{max} = 0.30 in, as follows:

$$\Delta_{allow} = \frac{L}{600} = \frac{(12.67)(12 \text{ in/ft})}{600} = 0.253 \text{ in}$$

$$\Delta_{allow} < \Delta_{max}; \text{ Use } \Delta_{allow} = 0.253 \text{ in}$$

$$\Delta_{allow} = 0.253 \text{ in} > \Delta_m = 0.242 \text{ in}$$

...Deflection O.K.

Bearing Check.

(a) Determine the maximum bearing stress, $f_{brg(max)}$, assuming a triangular stress distribution as follows:

$$f_{brg(max)} = \frac{V_{max}}{A_{brg}}$$

Where:

$$A_{brg} = 8 \text{ in} \times 7.62 \text{ in} = 61 \text{ in}^2$$

$$f_{brg(max)} = \frac{2(5361)}{61} = 176 \text{ lb/in}^2$$

(b) Determine the allowable bearing stress, F_{brg}, as follows:

$F_{brg} = 0.25(f'_m) = 0.25(1350 \text{ lb/in}^2) = 337 \text{ lb/in}^2$

$F_{brg} = 337 \text{ lb/in}^2 > f_{brg(max)} = 176 \text{ lb/in}^2$

...Bearing O.K.

(4) *Summary.* The 8-inch × 24-inch CMU lintel reinforced with 2-#14 bars is sufficient.

c. Design example 3.

(1) *Given.*

(a) 8-inch CMU loadbearing wall
(b) Wall height = 12 ft
(c) Wall panel length = 30 ft
(d) Uniform roof live load (w_{LL}) = 600 lb/ft
(e) Uniform roof dead load (w_{DL}) = 150 lb/ft
(f) Type S mortar
(g) f'_m = 1350 psi
(h) $f'_m = 1/3f'_m$ = 450 psi
(i) $E_m = 1000f'_m$ = 1,350,000 psi
(j) Reinforcement:
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000
(k) $n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$

(1) Two doors, 12-feet wide by 10-feet high, are located in the wall as shown in figure 8-6.

(2) *Problem.* Design the lintel over the doors.

(3) *Solution.* The three masonry courses above the opening will be solidly grouted and used as the lintel. The masonry lintel will be analyzed as a braced frame member. The ACI 318 moment and shear coefficients will be used to determine the approximate design moments and shears.

Flexure Check. The masonry frames meet all of the requirements of ACT 318 which allows the use of the approximate method. Determine the maximum moment envelope using the ACT 318 moment coefficients.

(a) Determine the maximum negative moment at the face of the interior support, M_{si}, as follows:

$$M_{si} = \frac{w_{tot}L^2}{9}$$

Where:

$w_{tot} = w + w_{DL} + w_{LL}$

And;

w = The lintel weight, lb/ft
= (92 psf)(2 ft) = 184 lb/ft
w_{tot} = 184 + 150 + 600 = 934 plf

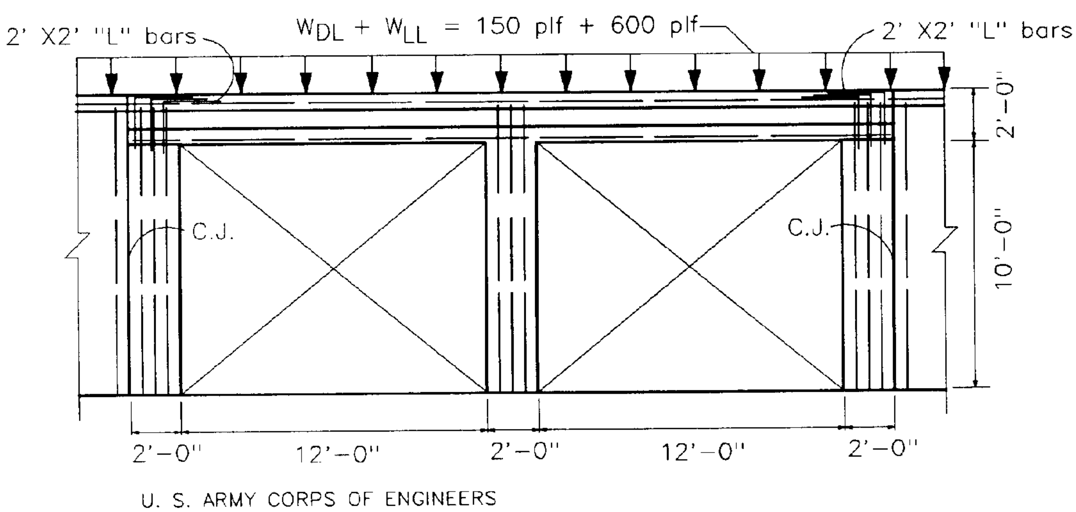

Figure 8–6. Example 3 wall elevation.

$$M_{si} = \frac{(934)(12\text{ ft})^2}{9} = 14{,}944\text{ ft-lb}$$

(b) Determine the maximum negative moment at the interior face of the exterior support, M_{se}, as follows:

$$M_{se} = \frac{wL^2}{16} = \frac{(934)(12)^2}{16} = 8{,}406\text{ ft-lb}$$

(c) Determine the maximum positive moment, M_{sm}, as follows:

$$M_{sm} = \frac{wL^2}{14} = \frac{(934)(12)^2}{14} = 9{,}607\text{ ft-lb}$$

Note. The maximum negative moment at the face of the interior support, M_{si} = 14,944 fl-tb, governs the flexural design.

(d) Determine the area of reinforcement, A_{sb}, required to provide a balanced steel ratio, P_e.

$A_{sb} = (p_e)(b)(d)$

Where:

$p_e = 0.0027$ (See table 5-9)
b = The lintel width = 7.62 in
d =The effective depth of lintel = 20.62 in
$A_{sb} = 0.0027(7.62)(20.62) = 0.424\text{ in}^2$

(e) The minimum reinforcing steel required above any wall opening is 1-#4 bar, $A_s = 0.20\text{ in}^2$.

$A_s = 0.20\text{ in}^2 < A_{sb} = 0.424\text{ in}^2$

Try 2-#4 bars ($A_s = 0.40\text{ in}^2$) as shown in figure 8-7.

(f) Obtain the masonry resisting moment, M_{rm}, and the reinforcing steel resisting moment, M_{rs}, from table C-9:

M_{rm} = 15,448 ft-lb
M_{rs}= 14,954 ft-lb

Note. The reinforcing moment, M_{rs}, governs the design.

M_{rs} = 14,954 ft-lb > M_{si}, = 14,944 ft-lb

...Flexure O.K.

(g) The steel reinforcement detailing should be as described below and shown in figure 8-6. Since the reinforcement in the top of wall bond beam is required to be continuous, as a diaphragm chord, bar cutoffs locations need not be considered. Frame action must be maintained at the corners, so corner bars will be used.

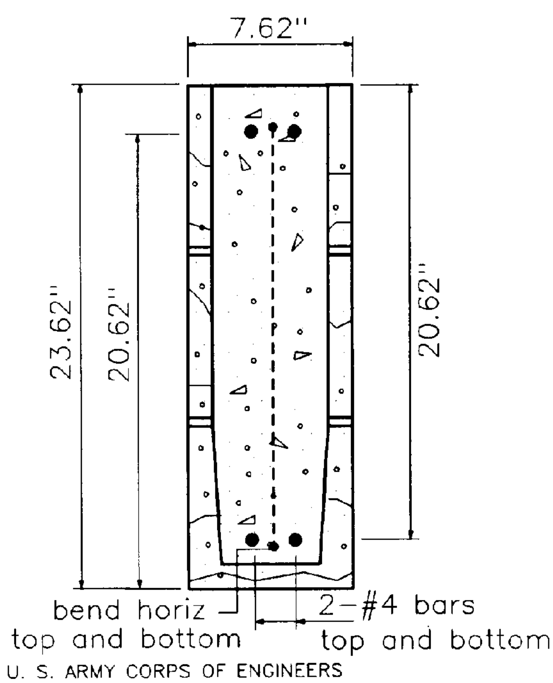

Figure 8-7. Example 3 cross section.

Deflection Check.

(a) The effective moment of inertia, I_e, is determined as follows:

$I_e = (M_{cr}/M_{max})^3 I_g + [1 - (M_{cr}/M_{max})^3] I_{cr}$

Where:

$$M_{cr} = \frac{f_r I_g}{Y_t}$$

And;

$f_r = 2.5\sqrt{f'm} + 2.5\sqrt{1350}$ lb/in^2 = 91.8 lb/in^2

$I_g = (7.62)(23.62)^3/12 = 8368$ in^4

$Y_t = 23.62/2 = 11.81$ in

$$M_{cr} = \frac{(91.8)(8368)}{11.81(12 \text{ in/ft})} = 5420 \text{ ft-lb}$$

I_{cr} = The moment of inertia of the cracked section, in^4

I_{cr} = 2,393 in^4 (From table C-9)

M_{max} = The maximum applied moment, ft-lb. (Use the average of the maximum negative moment, M_{si}, and the maximum positive moment, M_{si}, in computing I_e.

$$M_{max} = \frac{14944 + 9607}{2} = 12276 \text{ ft-lb}$$

$$I_e = \left[\frac{5420}{12276}\right]^3 (8368) + \left[1 - \left[\frac{5420}{12276}\right]^3\right](2383) = 2889 \text{ in}^4$$

$I_e = 2899\text{in}^4 < I_g == 8368$ in^4; Therefore use I_e in the deflection equations.

(b) The maximum total lintel deflection, Δ_{mt}, occurs when both spans are loaded with the uniform dead load and one span is loaded with the uniform live load and is determined as follows:

$\Delta_{mt} = \Delta_{ms} + (3)(\Delta_{ml})$

Since the long term dead load deflection, Δ_{ml}, is increased by a factor of 3, the dead and live load deflections will be determined separately.

(c) Determine the long term (dead load) deflection, Δ_{ml}, as follows:

$$\Delta_{ml} = (3)\frac{5L^2}{48EI}\left[M_{sm} - \frac{1}{10}(M_{se} + M_{si})\right]$$

Note. This equation was derived using the conjugate beam method and is the general expression for the elastic mid-span deflection for a uniformly loaded span with unequal end moments. The shear and moment diagrams for the load case that produces maximum dead load deflection are shown in figure 8-8.

Where:

M_{sm} =The positive moment at mid-span, ft-lb

$$= M_{max} - \frac{w(0.049L)^2}{2} = 0.0377wL^2$$

And;

M_{max} = The maximum positive moment in the span, ft-lb

$$= -\frac{wL^2}{16} + \frac{w(0.451L)^2}{2} = 0.0389 \text{ wL}^2$$

$$M_{sm} = 0.0389wL^2 - \frac{(0.049L)^2 w}{2} = 0.0377wL^2$$

$$= 0.0377(33)(12)^2(12 \text{ in/ft}) = 21{,}758 \text{ in-lb}$$

$$M_{se} = \frac{1}{16}(334)(12)^2(12 \text{ in/ft}) = 36{,}072 \text{ in-lb}$$

$$M_{si} = \frac{1}{9}(334)(12)^2(12 \text{ in/ft}) = 64{,}064 \text{ in-lb}$$

$$\Delta_{ml} = (3)\left[\frac{5(12)^2(144 \text{ in}^2/\text{ft}^2)}{48(1{,}350{,}000)(2899)}\right] \times \left[21{,}758 - \frac{1}{10}(36{,}072 + 64{,}062)\right] = 0.018 \text{ in}$$

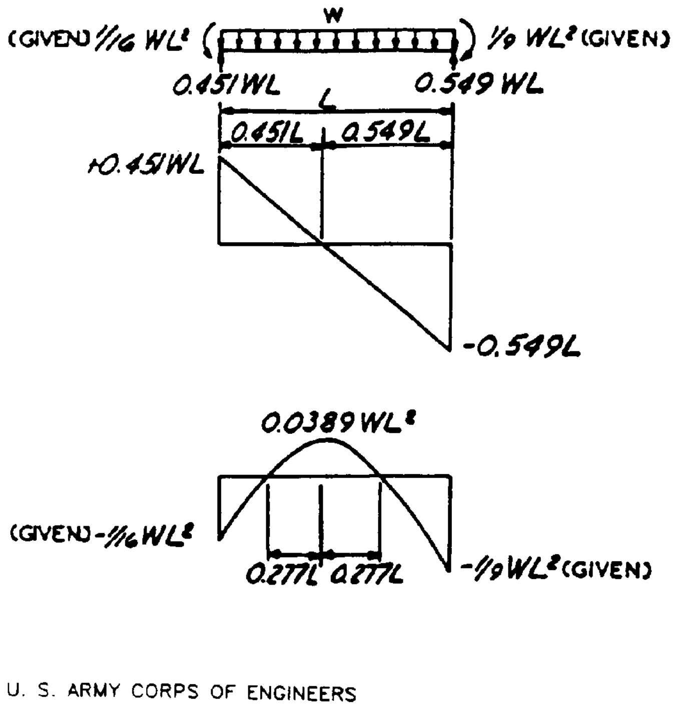

Figure 8–8. Dead load shear and moment diagrams.

(d) Determine the short term (live load) deflection, Δ_{ms}, as follows:

$$\Delta_{ms} = \frac{5L^2}{48EI}\left[M_{sm} - \frac{1}{10}(M_{se} + M_{si})\right]$$

Note. The shear and moment diagrams for the load case that produces maximum live load deflection are shown in figure 8-9.
Where:

$$M_{sm} = -\ 0.03wL^2 + \frac{(0.451L)^2w}{2} - \frac{(0.049L)^2w}{2}$$
$$= 0.072wL^2 = 0.071(600)(12\text{ ft})^2(12\text{ in/ft})$$
$$= 73{,}613\text{ in-lb}$$

Note. M_{max} was assumed to be located at the same point on the beam as determined from the dead load analysis. This is an approximate method of determining the shear and moment diagrams, and is reasonably accurate. The designer may decide to make another reasonable assumption or use a more accurate method of analysis, but the difference in the results will be small.

M_{se}= 0.03(600)(12)^2(12 in/ft) = 31,104 in-lb
M_{si} =0.079(600)(12)^2(12 in/ft) = 81,907 in-lb

$$\Delta_{ms} = \left[\frac{5(12)^2(144\text{ in}^2/\text{ft}^2)}{48(1{,}350{,}000)(2899)}\right]$$
$$\times\left[73{,}613 - \frac{1}{10}(31{,}104 + 81{,}907)\right] = 0.034\text{ in}$$

Δ_{mt} = 0.018 in + 0.034 in = 0.052 in

(e) Determine the allowable lintel deflection, Aaiiow, and compare to the maximum lintel deflection, Δ_{max} = 0.30 in, as follows:

$$\Delta_{allow} = \frac{L}{600} = \frac{(12)(12\text{ in/ft})}{600} = 0.24\text{ in}$$

$$\Delta_{allow} < \Delta_{max};\ \text{Use } \Delta_{allow} = 0.24\text{ in}$$

$$\Delta_{allow} = 0.24\text{ in} > \Delta_{mt} = 0.052\text{ in}$$

...Deflection O.K.

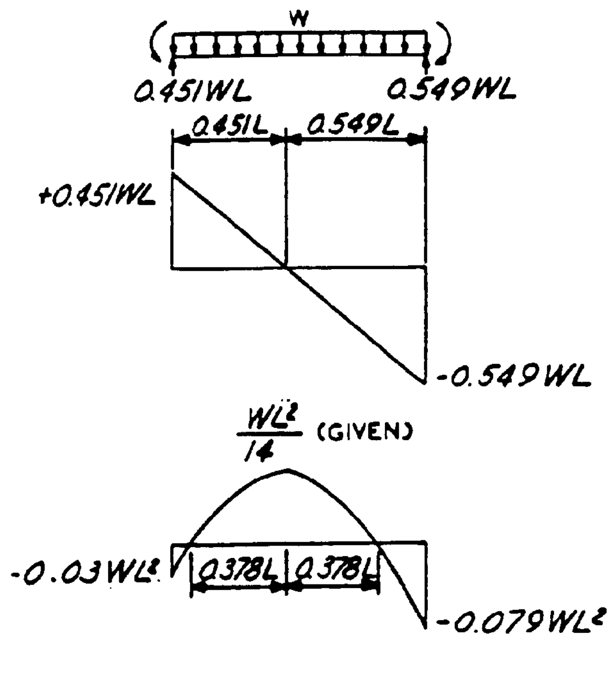

Figure 8–9. Live load shear and moment diagrams.

Shear Check.

(a) Determine the shear stress in the lintel, F_v, using the ACI shear coefficients as follows:

$$f_v = \frac{V}{bd}$$

The shear stress at the face of the first interior support, f_{vi}, is determined as follows:

$$f_{v1} = \frac{V_1}{bd}$$

Where:

V_1 = The shear force at the first interior support, lb
= 1.l5wL/2= 1.15(934)(12)/2 = 6445 lb

$$f_{v1} = \frac{V_1}{bd} = \frac{6445}{(7.62)(20.62)} = 41.02 \text{ lb/in}^2$$

The shear stress at the face of the exterior support, f_{v2},is determined as follows:

$$f_{v2} = \frac{V_2}{bd}$$

Where:

V_2 = The shear force at the exterior support, lb
= wL/2 = (934)(12)/2 = 5604 lbs

$$f_{v2} = \frac{V_2}{bd} = \frac{5604}{(7.62)(20.62)} = 35.7 \text{ lb/in}^2$$

Note. The shear stress at the first interior support governs the design.
So; $f_v = f_{v1} = 41.02$ lb/in^2

(b) Determine the allowable shear stress, F_v, as follows:
$F_v = 1.0\sqrt{f'_m} = \sqrt{1350}$ lb/in^2 = 36.7 lb/in^2

f_v =41.02 psi > F_v = 36.7 psi; Therefore, shear reinforcement is required in the beam at the interior support. Since f_v > 20 psi and there is required negative reinforcement, web reinforcement must be provided to carry the entire shear for a distance of one-sixteenth the clear span beyond the point of inflection. The allowable shear load based on 20 psi, V_{allow}, is:

V_{allow} = (20)(7.62)(20.62) = 3142 lb

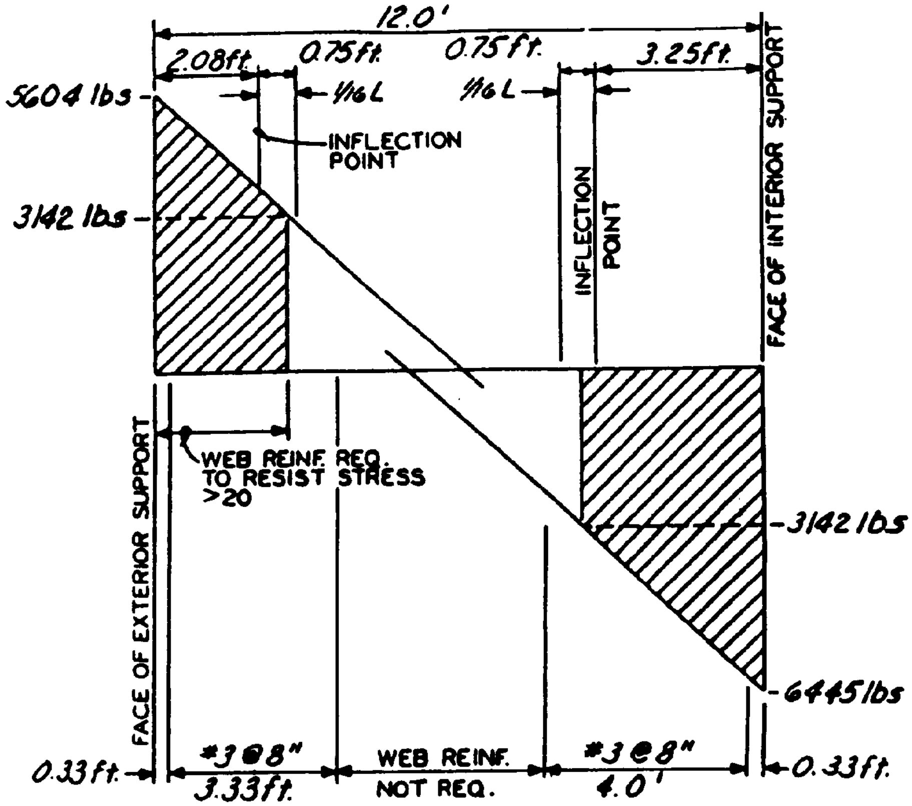

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Figure 8–10. Location of web reinforcement in lintel.

V_2 = 5604 lb > V_{allow} = 3142 lb; Therefore, web reinforcement is required at both ends of the span.

(c) The area of the shear reinforcement required, A_v, is determined as follows:

$$A_v = \frac{V_1 s}{F_s d}$$

Where:

F_s = The allowable stress in the steel, psi
= 24,000 psi

s = The spacing of the shear reinforcement, in inches. The spacing of shear reinforcement should not exceed d/2 nor 24 inches. The maximum spacing, s_{max}, of the shear reinforcement is:

s_{max} = d/2 = 20.62 in/2 = 10.31 in

Use s = 8 in (modular in reinforcement CMU)

$$A_v = \frac{(6445\ \text{lb})(8\ \text{in})}{(24{,}000\ \text{psi})(20.62)\ \text{in}} = 0.104\ \text{in}^2$$

Use 1-#3 bar (A_v = 0.11 in^2), with web reinforcement provided starting at 4 inches from the face of the support, spaced at 8 inches on center, to the first 8-inch module beyond the inflection point plus one-sixteenth of the span as shown on figure 8-10.

(d) When all of the shear is resisted by the reinforcement the maximum allowable shearing stress, Max F_v, must be checked as follows:

Max $F_v = 3\sqrt{f'_m}$ not to exceed 120 psi
$= 3\sqrt{1350}$ lb/in^2 = 110 lb/in^2 < 120 lb/in^2

Use Max F_v = 110 lb/in^2

f_v = 41.02 lb/in^2 < Max F_v = 110 lb/in^2

...Shear O.K.

(e) Since the top of the lintel is the top of the wall, the top face compression region of the lintel must be given lateral support. The maximum distance between points of lateral support, l_c, is determined as follows:

$$1_c = 32(b) = 32(7.62) = 244 \text{ in} = 20.33 \text{ ft}$$

(4) *Summary.* The 8-inch by 24-inch CMU lintel with 2-#4 bars top and bottom and 1-#3 @ 8” o.c. shear reinforcement located as shown in figure 8-10 is sufficient. The top of the lintel must be laterally supported at a maximum spacing of 20 feet.